Chapter 4
Loads and Loading Conditions

4-1. General

Structure dead loads and backfill loads generally do not vary and are a basic part of the usual, unusual, and extreme load conditions used to evaluate stability. The loads that vary are those due to water pressures and those associated with earthquakes. The hydrostatic pressures on structures that occur during normal operation differ from those occurring during maintenance unwatering, floods, and rapid drawdown events. Loads for earthquake events likely to occur during the lifetime of the structure are different from those loads generated by maximum credible earthquakes. Paragraphs 4-2 through 4-4 describe in general terms the types of water loads and earthquake loads associated with usual, unusual, and extreme load conditions. Engineers performing stability analyses are required to investigate all possible loads and loading conditions and to determine which control the design. Paragraphs 4-5 through 4-7 provide information on uplift loads, earthquake loads, and other miscellaneous loads such as impact, wind, ice, debris, and hawser pull. Methods for applying calculated loads associated with multiple- and single-wedge analyses are covered in Chapter 2 and Chapter 5, respectively.

4-2. Water Loading Conditions

a. Water loads - general. Water loads that fall into the usual load condition category include those maximum differential head and uplift conditions associated with normal operation and flood events that have return periods of 2 years or less. The unusual load condition category includes those maximum differential head and uplift conditions associated with events that have return periods greater than 2 years but less than or equal to 150 years. Another way of defining the upper limit of the unusual load condition category is to say that the pool levels, or flow lines for channel projects, correspond to maximum differential head and uplift conditions that have a 50 percent chance of being exceeded during a 100-year service life. The extreme-load condition category includes those maximum-differential head and uplift conditions associated with flood events having return periods greater than 150 years, up to the probable-maximum flood. Extreme load conditions are also assigned to water loads associated with loss of pool or rapid drawdown events. Water loads for unusual, or extreme loading conditions are only to be combined with other loads that occur during routine operation.

b. Normal operation. In the past, a normal-operation loading condition has been used to describe loadings with various probabilities of occurring, including rare events with long return periods. To be consistent with Table 3-1, normal operating conditions are now defined as maximum loading conditions with a return period of no more than 2 years. For certain floodwalls, this means that there are no water loads on the structure for normal operation. For hydropower dams, the pool will be fairly high for normal operation, while for some flood-control dams, the pool will be low for the 0.5 annual probability event which represents normal operation.

c. Infrequent flood. The infrequent flood (IF) is a general term used to describe selected loading conditions traditionally examined for stability. It usually represents a loading of large magnitude, but less than the maximum the structure may experience. The IF is related to some physical dimension of the project. For dams with ungated spillways, the IF is when water is at the spillway crest elevation. For dams with gated spillways, the IF is when water is at the top of the closed gates. For inland floodwalls, the IF is when water is 0.6 m (2 ft) below the top of the wall. This approximates the old nominal design water surface (i.e., top-of-wall minus freeboard). IF loadings are included in the tables in Appendix B. The IF loading will be classified as usual, unusual, or extreme per Table 3-1, based on its return period, though normally the IF will be an unusual event.

d. Probable maximum flood. The probable maximum flood (PMF) is one that has flood characteristics of peak discharge, volume, and hydrograph shape that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydrometerological analyses of critical runoff-producing precipitation, snow melt, and hydrologic factors favorable for maximum flood runoff. The PMF load condition represents the most severe hydraulic condition, but because of overtopping and tailwater effects, it may not represent the most severe structural loading condition, which is represented by the maximum design flood described below. Therefore, the PMF condition will not necessarily be examined for structural stability.

e. Maximum design flood. The maximum design flood (MDF) is the designation used to represent the maximum structural loading condition and must be determined for each structure or even for each structural element. MDF may be any event up to PMF. For floodwalls, MDF is usually when the water level is at or slightly above the top of the wall. Overtopping from higher water levels would result in rising water levels on the protected side, thus reducing net lateral forces. The same situation may be true for dams, but often significant overtopping can occur without significant increases in tailwater levels. The design engineer must consult with the hydraulics engineer to explore the possible combinations of headwater and tailwater and their effects on the structure. Some elements of dam outlet works (such as chute walls or stilling basins) are loaded differently from the main dam monoliths. For such elements, different flow conditions will produce maximum structural loading. When it is not obvious which loading will produce the lowest factor of safety, multiple loadings should each be investigated as a possible MDF. Since sliding is the most likely mode of failure for most gravity structures, MDF can usually be judged by determining maximum net shear forces. However, due to variable uplift conditions, a loading with smaller shears could result in the lowest factor of safety. Once the MDF is determined, it should be classified as usual, unusual, or extreme per Table 3-1, based on its return period.

4-3. Earthquake Loading Conditions

Earthquake-generated inertial forces associated with the operational basis earthquake (OBE) are considered to be unusual loads. Those associated with maximum design earthquake (MDE) or those associated with the maximum credible earthquake (MCE) are considered extreme loads. Earthquake loads are to be combined with other loads that are expected during routine operations.

a. Operational basis earthquake. The OBE is considered to be an earthquake that has a 50 percent chance of being exceeded in 100 years, (or a 144-year return period).

b. Maximum design earthquake. The MDE is the maximum level of ground motion for which a structure is designed or evaluated. Generally, the probabilistically determined MDE for normal structures is a earthquake that has a 10 percent chance of being exceeded in a 100-year period, (or a 950-year return period). For critical structures the MDE is the same as the MCE.

c. Maximum credible earthquake. The MCE is defined as the greatest earthquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geological evidence. The MCE is based on a deterministic site hazard analysis.

4-4. Maintenance Conditions

The stability analyses procedures described herein were formulated on the basis of maintenance loads being unusual loads. The return periods for a maintenance condition loading may be less than 2 years, but based on past experience the designation of maintenance load conditions as unusual is acceptable. The use of the unusual-load-condition category is based on the premise that maintenance load conditions only take place under controlled conditions, and that the risks are only taken during the short time the structure is unwatered.

4-5. Uplift loads

a. General. Uplift loads have significant impact on stability. Sliding stability, resultant location, and flotation are all aspects of a stability analysis where safety can be improved by reducing uplift pressures. Uplift pressures are directly related to flow paths beneath the structure, and uplift pressure heads at points of interest can be obtained from a seepage analysis. Such an analysis must consider the types of foundation and backfill materials, their possible range of horizontal and vertical permeabilities, and the effectiveness of cutoffs and drains. Techniques of seepage analysis are discussed in EM 1110-2-1901, Casagrande (1937), Cedergren (1967), Harr (1962), and EPRI (1992). Seepage techniques applicable to uplift pressure determination on structures include flow nets, numerical methods such as the finite element method, the line-of-creep method, and the method of fragments. Uplift pressures can be reduced through foundation drainage, or by various cutoff measures such as grout curtains, cutoff walls, and impervious blankets. The effects of drainage and cutoffs are described in the following paragraphs. Uplift pressures to be used for the stability design of various types of new structures are covered in Appendix C. The conservative uplift pressures used for the design of new structures may be significantly higher than those the actual structure may experience during its lifetime. For this reason, the use of actual uplift pressures for the evaluation of existing structures is permitted under the provisions discussed in Chapter 6.

b. Effect of drainage on uplift pressures. Drainage is an effective means of reducing uplift pressures for structures founded on rock. It also is effective for structures founded on soils, provided the loss of soil materials through piping can be prevented. Uplift pressures under a gravity dam, with and without-drains, is illustrated in Figure 4-1. The without-drains uplift-pressure diagram is in the general shape of an S-curve which is commonly replaced with a straight line for use in stability analyses. The actual uplift diagram takes on the shape of an S-curve instead of being straight because of the tendency of the seepage flow to follow the shortest path and thus crowd close to both the heel and toe, increasing velocity and, therefore, the rate of loss of head at these locations. For the with-drains condition, uplift downstream of the line of drains will be at or near tailwater as shown in
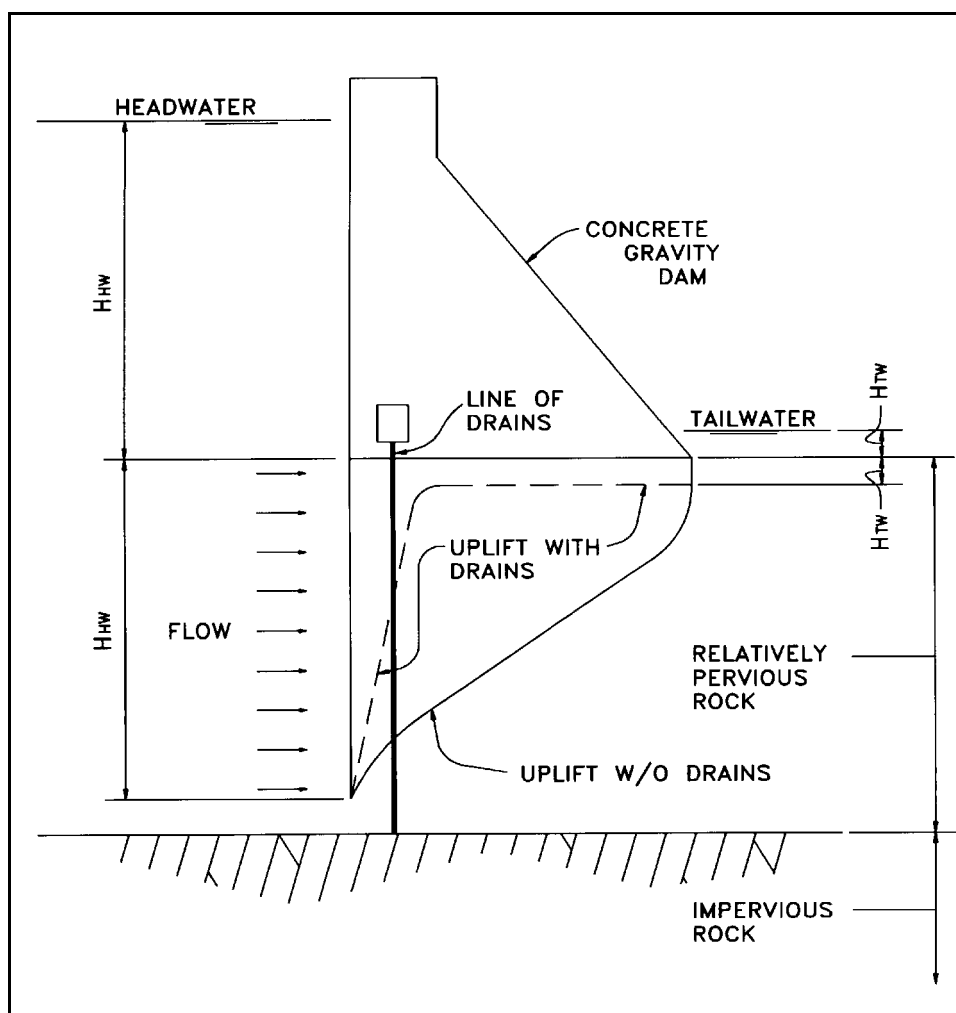

Figure 4-1.   Drain effect on uplift pressures

Figure 4-1, provided the drainage gallery floor is at or below tailwater, and provided the drains penetrate the pervious strata. This assumes that seepage flows enter the drains through a vertical entrance face at the heel of the dam. Figure 4-1 illustrates a drain efficiency of 100-percent. For design, it would be unconservative to assume that the aforementioned ideal drainage conditions exist. Normally for design purposes, the drain efficiency is assumed to be 50 percent. In many existing dams, however, it is common to find uplift pressures downstream of the drains at, or slightly above, tailwater. On occasion, some designers will check stability of an existing structure for a drains-clogged condition. Since drainage is such an important factor in reducing uplift pressures, the best policy is to regularly inspect, maintain, and clean the drains to prevent clogging. In cases where it is impossible to clean the drains, a drains-clogged condition should be included as part of the stability analysis for an existing structure. This load condition should be treated as a usual, unusual, or extreme load condition depending on the return period for the load condition being evaluated.

c. Effect of cutoffs on uplift pressures. Cutoffs are also effective in reducing uplift below structures. The effectiveness of cutoffs, however, can be jeopardized by leakage through joints, cracks, and fractures. Therefore, drains are considered to be the most reliable and cost-effective way of reducing foundation-uplift pressures, especially for structures founded on rock. Although grout curtain cutoffs are commonly used in combination with drainage systems for dams founded on rock, the grout-curtain cutoff helps more to reduce drain flows in the drainage gallery than to reduce uplift pressures. Cutoffs can be either grout curtains, concrete trenches, steel sheet piling, or impervious blankets. The uplift-reduction benefits of a sheet-pile cutoff, driven to a depth equal to the structure base width, is shown in Figure 4-2. This figure represents the case of a gravity dam founded on sand. For a discussion on the effectiveness of sheet-pile cutoff walls, see Chapter 5.

4.6 Earthquake Loads

a. General. Analytical methods are available to evaluate the dynamic response of structures during earthquakes, i.e., seismic coefficient methods, response spectrum methods, and time-history methods. These methods are discussed in reference ER 1110-2-1806. The current state-of-the-art method involves the use of linear elastic and nonlinear finite element time history analysis procedures which account for the dynamic interaction
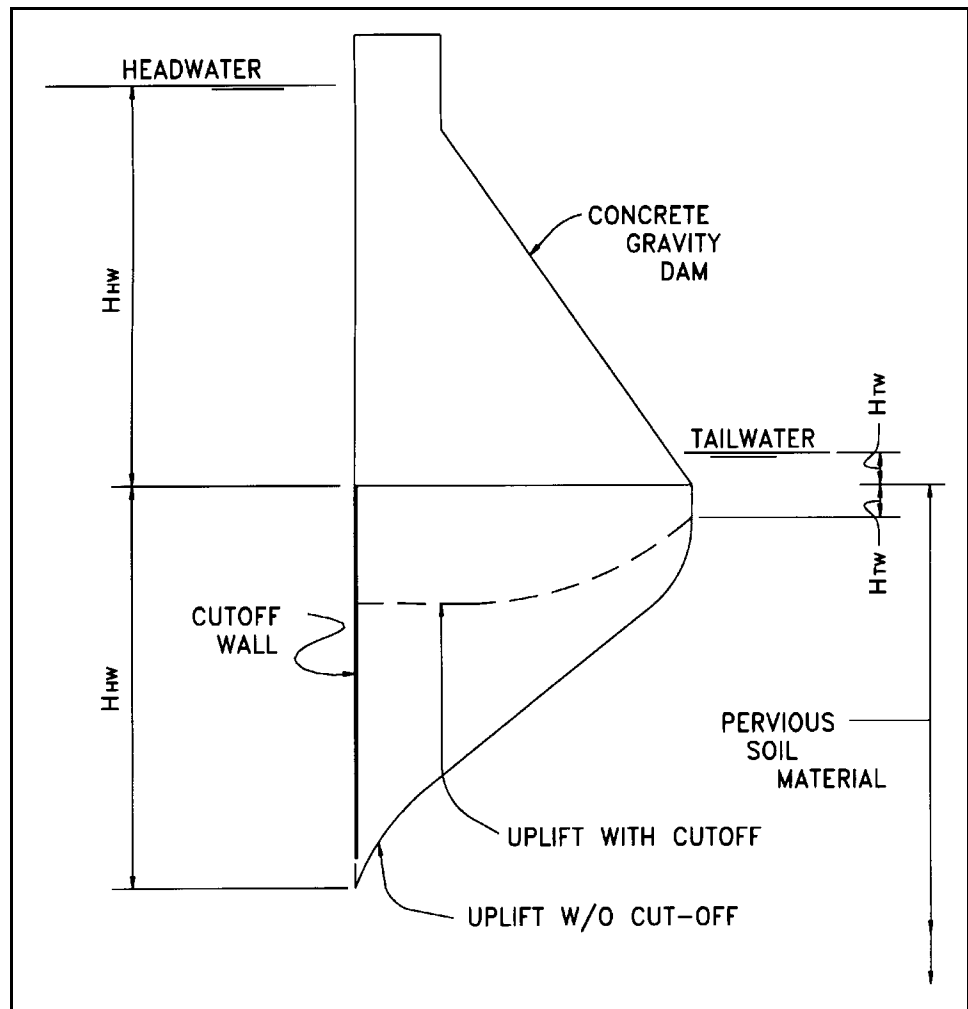

Figure 4-2. Cutoff effect on uplift pressure

between the structure, foundation, soil, and water. Traditional seismic coefficient methods are presented in Corps manuals for use in evaluating the stability of structures that may be subjected to earthquake ground motions. The seismic coefficient method, although it fails to account for the true dynamic characteristics of the structure-water-soil system, is sometimes accepted as the method for evaluating structural stability, and is often used as a tool to decide if dynamic analyses should be undertaken. The information in the following paragraphs describes the differences between the seismic coefficient method and dynamic analysis methods. Figure 4-3 illustrates the differences in the inertial and hydrodynamic earthquake loads obtained by the two different methods. As can be seen from the figure, the magnitude and distribution of earthquake loads is quite different depending on whether dynamic analysis methods are used to obtain the loads, or the seismic coefficient method is used. Earthquake loads are used to represent the inertial effects attributable to the structure mass, the surrounding soil (dynamic earth pressures), and the surrounding water (hydrodynamic pressures). In the traditional seismic coefficient approach, it is customary to use accelerations that are equal to or less than the peak ground accelerations expected during the design earthquake event. Actually, portions of the structure may experience accelerations considerably greater than the peak ground acceleration. Inherent in the seismic coefficient approach is the assumption that the structure will undergo some acceptable permanent displacement should a major earthquake occur.
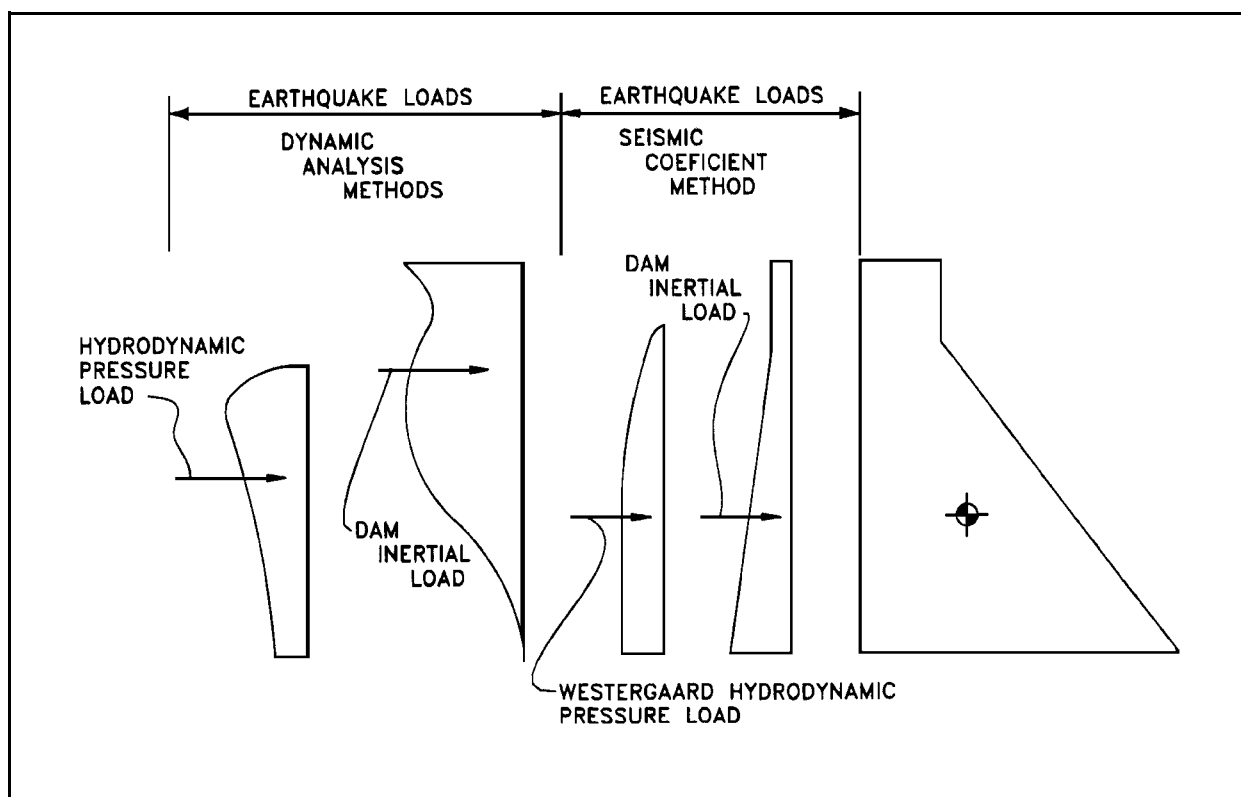

Figure 4-3. Dynamic analysis method versus Seismic coefficient method

   b.   Inertial effects - structure mass. In the traditional seismic coefficient approach, the inertial force is computed as the product of the structure mass and the seismic coefficient expressed as a fraction of gravity. The force is located so as to act in a horizontal direction at the center of mass of the structure, based on the assumption that the structure is a rigid body. In actuality, almost all structures have some flexibility, and the use of the rigid body concept often under estimates the magnitude of the inertial force. The location of the inertial force is also related to the flexibility of the structure, and usually the force acts at a location higher than the center of mass. However, because of the cyclic nature of earthquake loads, there is little opportunity for a rotational-stability related failure.

c. Inertial effects - surrounding water. Water adjacent to, or surrounding a structure will increase inertial forces the structure will experience during an earthquake. As the structure is displaced during an earthquake, it pushes through the surrounding water thereby causing hydrodynamic forces to occur on the structure. The water inside and surrounding the structure alters the dynamic characteristics of the structural system, increasing the fundamental mode of vibration and modifying the mode shapes. In the traditional seismic coefficient methods, the hydrodynamic effects are simplified. For dams, navigation lock walls, and retaining walls, the hydrodynamic effects are usually approximated using the Westergaard Formula. This method assumes the structure is rigid and the water is incompressible. Since most structures are flexible, the Westergaard Formula can lead to significant error. For free-standing intake towers, the hydrodynamic effects are approximated by adding mass to the structure to represent the influence of the water inside and surrounding the tower. The Westergaard approach is described in EM 1110-2-2200. Engineers using the seismic coefficient approach for stability analyses should be aware of the limitations of the approach and the simplifying assumptions made with respect to hydrodynamic pressures and their distribution on the structure.

d. Inertial effects - surrounding soil. Backfill material or soil adjacent to a retaining wall will induce inertial forces on the wall during an earthquake. These inertial forces are estimated using the Mononobe-Okabe Equation. This procedure is described in Chapter 5.

4-7. Other Loads

Other loads to be considered in stability analyses are loads due to impact, ice, debris, and hawser pulls.

a. Impact. Impact loads for locks and dams on navigation systems are due to the structures being struck by barges. These loads can be quite large and for some structures, such as lock guide walls, control the stability analyses. Where impact loads must be considered, refer to EM 1110-2-2602.

b. Ice. Loads due to ice are usually not critical factors in the stability analysis for hydraulic structures. They are more important in the design of gates and other appurtenances. Ice damage to gates is quite common, but there is no known case of a dam failure due to ice. Where ice loads must be considered, refer to EM 1110-2-1612.

c. Debris. Debris loads, like ice loads, are usually of no consequence in stability analyses. However, they may be critical for the design of gates and floodwalls.

d. Hawser pull. Hawser pulls are significant in the stability analysis for lock guide walls, mooring facilities, and flood walls. Where hawser pulls must be considered, refer to EM 1110-2-2602.

e. Wind. Wind loads are usually small in comparison to other forces which act on Civil Works structures. Therefore, wind loads should not usually be combined with other forces acting on a completed structure. For structures such as coastal flood walls where wind might cause instability, or for structures under construction, wind pressures should be based on the requirements of ASCE 7-88.

4-8. Loading Condition Summaries

Previously, stability criteria was provided in the Engineer Manuals listed in Appendix A. The load cases from those manuals are summarized in tables provided in Appendix B. The original load case number, a description of the load case, and a classification of usual, unusual, or extreme is provided. Often the older design guidance did not describe the flood conditions in terms of the MDF, or the PMF, nor did it describe the earthquake loads in terms of the OBE, the MDE, or the MCE. In order to bring the older guidance up-to-date with this manual and other recent manuals, the original load cases from the older manuals have been revised to comply with the terminololgy used in this manual.